DELEUW

# Problems in Design and Construction of Large Land Drainage Districts

Civil Engineering

B. S.

1912

# PROBLEMS IN DESIGN AND CONSTRUCTION OF LARGE LAND DRAINAGE DISTRICTS

BY

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**THESIS**

FOR THE

DEGREE OF BACHELOR OF SCIENCE

IN

CIVIL ENGINEERING

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COLLEGE OF ENGINEERING

UNIVERSITY OF ILLINOIS

1912

UNIVERSITY OF ILLINOIS

COLLEGE OF ENGINEERING.

May 24, 1912

This is to certify that the thesis of CHARLES EDMUND DeLEUW entitled PROBLEMS IN DESIGN AND CONSTRUCTION OF LARGE LAND DRAINAGE DISTRICTS was prepared under my personal supervision; and I recommend that it be approved as meeting this part of the requirements for the degree of Bachelor of Science in Civil Engineering.

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## TABLE OF CONTENTS.

Introduction Page 1

Chapter I, Levees.

1. Determination of Height. 3
2. Determination of Cross-Section. 4
3. Location and Construction. 5

Chapter II, Run-Off.

1. Formulas 7
2. Computation of Run-Off and Comparison with Actual Discharge. 9
   - A. Lower Salt Fork Drainage District. 9
   - B. North Fork Special Drainage District. 10
   - C. P. C. Knight's Examples. 11
   - D. Pekin and Lamarsh Drainage District. 12
   - E. Red River Valley in North Dakota. 12
   - F. Cook Ditch in Kankakee River Valley. 14
3. Run-Off as Affecting Design. 17
   - A. Ditches. 17
   - B. Pumping Plant. 17

Chapter III, Ditches.

1. Formulas for Discharge of Ditches. 20
2. Cross-Section of Ditches. 21
3. Slope of Ditches. 23
4. Velocity in Ditches. 24
5. Location and Construction of Ditches. 26

Chapter IV, Tile Drains.

1. Duty of Tile Drains Determines by Run-Off. Page 27
2. Spacing and Depth of Tile Drains. 28
3. Formulas for Discharge of Drains. 31
4. Slope of Tile Drains. 32
5. Location and Construction of Tile Drains. 33

Conclusion. 34

# PROBLEMS IN DESIGN AND CONSTRUCTION OF LARGE LAND DRAINAGE DISTRICTS.

## Introduction.

The reclamation of swamps and overflow lands has been going on for years. From data collected and analyzed by the United States Department of Agriculture, and published in Circular No. 76 of that department, it is certain that there are, in the portion of the land East of the Rocky Mountains, 77,000,000 acres that can be reclaimed and made fit for cultivation by building simple engineering structures. A large part of this land is in the Mississippi River valley. In Illinois there are 2320 square miles and in the whole valley there are, at a conservative estimate, about 65,000 square miles. On this land, in the river bottoms, the overflow of streams has been depositing silt for ages. This has formed a rich, grey, alluvial soil, which on becoming mixed with decomposed vegetable matter becomes a dull black. This is the composition of the lands in these bottoms, except for an occasional sand ridge, and accounts for the fact that they are the most fertile and productive lands in the country, when properly drained.

It is the purpose of this paper to take up and discuss as fully as possible some of the problems arising in the design and construction of large land drainage districts or units. The first thing noted in the study of drainage work is the scarcity of good texts or reference works on the subject. It has also been

noted that of the articles and works which are available the major portion are of such a general nature that they are useless as an aid in the preparation of a thesis on this subject. In preparing this thesis the writer has endeavored to thoroughly look over all of the available works on drainage, and to record here the most important and rational of the ideas contained therein. Another striking fact which impresses one in looking over available literature on this subject is that engineers differ on many important and fundamental principles involved. This would seem to indicate that there have been very few tests made to bear out the different theories advanced. It is generally conceded that in the past scientific investigation has not kept pace with the expenditure of money for drainage purposes, and that there is a great need for tests along the lines of drainage work.

There have been millions of dollars wasted on drainage work in this country due to several different causes. First is the fact that competent engineers have had very little in the way of rational tests to base their designs upon, hence they have often had to learn by experience, which has proved very costly in many cases. Second, the drainage laws as they exist in many states are such as to make the administration of drainage districts very loose and uneconomical. Third, through carelessness of commissioners of districts or of land owners, the engineering work is often placed in the hands of some incompetent person, oftentimes in the hands of a person with no engineering knowledge whatsoever. This is often done because a farmer can get a tiler for a very small amount per day, and the idea has been quite general until

lately that the tiler could do about as good work without the engineer's aid as with it. This practice is detrimental to the land owner, as the failure of one drainage work not only wastes money in that particular case, but it may also, as an example, discourage other land owners in the immediate vicinity from undertaking meritorious projects. Many new and good drainage laws have recently gone into effect, and the farmers are now able to combine advantageously and with the new state of affairs they are beginning to see the economy of employing capable engineers.

## Chapter I. Levees.

### 1. Determination of Height.

The first step in the drainage of lands subject to overflow is the prevention of overflow in times of high water or floods. Probably fifty per cent of the bottom land in the Mississippi River valley falls under this class. The prevention of overflow is accomplished by means of construction of levees or dikes. The levee or dike is thrown up on the river sides of the district and is usually effective. In pumping districts, in some cases, the level of water inside the district is lower the year round than the surface of the river outside of the levee, the difference of elevation being maintained by pumping.

As the purpose of the levee is to exclude all flood waters, the height of the levee should be sufficient to exclude the highest water. The determination of this height is a very important factor in the success or failure of the whole design. This was shown in a very realistic manner, when many supposedly safe levees gave way during the recent flood of the Mississippi

River, causing millions of dollors of loss, and the loss of many lives. To determine the proper height, all available records of high water should be examined carefully, old residents should be consulted, and the high water marks on the timber should be looked over. To provide a margina of safety, the height of the levee should be such that its top will be at least three feet higher than the extreme high water mark. It should be borne in mind that when the levee is constructed the shape of cross-section in flood times is much changed, and due allowance should be made for the increase in depth due to narrowing the channel.

## 2. Determination of Cross-Sections.

The cross-section of the levee depends mainly on the nature of the soil, although the wave action should also be taken into consideration. No hard and fast rules can be laid down for the selection of a proper cross-section, as each individual case has peculiarities of its own, and the design will always call for the exercise of good judgment on the part of the engineer. It can be said, however, that with a heavy black soil, a levee with a 2-1/2 to 1 slope on the outside and a 1-1/2 to 1 slope on the land side, has been found to stand, where not subject to excessive wave action. A mixture of sand and clay should have an outer slope of about 4 or 5 to 1 and about 2 to 1 for an innerslope. The width of the crown of a levee is governed by local condition, such as method of construction, wave action, permeability of the soil, but it should under any conditions have a minimum width of six or eight feet. The berm width is governed by the same conditions.

Where there would be any liability of seepage underneath the

levee, it is thought necessary to construct a muck or base ditch. In any kind of soil this is a wise precaution, and the reduction of the amount of seepage will probably warrant the additional expense of constructing the muck ditch. This ditch should be from 2 to 4 feet wide and deep enough to cut through any sand or other pervious strata. The ditch can be built with teams and scrapers, a trench digger, or by hand. It should be filled with clay or some other impervious material.

### 3. Location and Construction.

The ground surface of the levee should be carefully prepared before any material is placed on it. The area to be covered by the levee should be cleared of all vegetation and the roots and stumps grubbed out to a depth of 3 feet or more. After grubbing the holes should be compactly filled up and tamped. The foundation should be plowed deep and thoroughly and in case a muck ditch is not dug, the foundation should be plowed outward so as to leave a deep, dead furrow in the center.

In the past much levee construction has been by means of teams and scrapers, but this method seems to be going out of general use. The best practice now seems to favor the construction of levees by dredges. It has been found that under normal conditions, dredges will construct a better and cheaper levee than is possible by any other means. In the construction of a levee of any size with a dredge, it is necessary to go over the levee several times as the wet material will not stand. This insures a closely compact levee as the material is deposited wet

and will run into a solid mass with small voids. Between each trip of the dredge the layer deposited the preceeding time has hardened, and as the surface is rough, there is an excellent bond between the different layers. A drag-line dredge has the advantage of being able to leave a greater berm than a dipper dredge. This is an important item as many failures of levees are due to a lack of a sufficient berm. The width of berm to use is a matter of judgment and depends upon the angle of repose of the soil, as has been stated before, but care should be taken to see that too small a machine is not used. A certain allowance for shrinkage should be made varying with the method of construction and nature of the soil. With dredge construction, the allowance can be small, but with a levee which is put up dry, with teams and slips, an allowance of ten or fifteen per cent should be made.

The levee location should be made with a view to taking the material from outside the levee, and should provide also sufficient slope between the toe of the slope and the edge of the borrow pit. As wave action is the most dangerous of all factors harmful to levees, a fore-shore covered with brush or timber, should be left or provided. Deep borrow pits are to be avoided, and as a general proposition, the wider the berm is made, the safer the levee will be. A covering of Bermuda grass on the levee will do much to protect it against erosion.

## Chapter II. Run-Off.

In order to discuss intelligently the subject of drainage, something of the movements of ground water should be known. It is known that water exists in soils in three different conditions: First, hygroscopic; second, capilliary; third, hydrostatic. All soils contain the first, it is the thin film that is held by the attractive force of the soil particle so firmly that it cannot be taken away by plants. When the soil is moist to any extent, the second form of water, or capillary water is present. It is held in the soil by surface tension and moves from one particle of soil to another by surface tension, the direction being towards the one which has the greatest tension, or from the more moist to the drier soil. This form of water is that used by all plants. It occupies only a part of the air space in the soil and under best conditions should occupy only about 50 per cent of the total water capacity of the soil. The third form of water, hydrostatic, should be drained from the soil to a depth of from 3 to 4 feet. The surface of this free water is called the water table or the ground water plane. This is the water which is detrimental to the crops and must be removed before the land can be cultivated.

### 1. Formulas.

From the foregoing it is obvious that in reclaiming wet lands, it is not only necessary that surface water be removed, but it is essential that this hydrostatic water should be taken off. The computation of run-off, however, does not include this hydro-

static water, but only the surface water. Several formulas have been advanced by different men; all of them are necessarily empirical in their nature. It is known that computation of run-off is a problem which admits of no exact mathematical solution. These formulas have been advanced for different situations and topographical lay-outs, and in their use, good judgment is a requisite. The formulas in most common usage are enumerated below.

Fanning's Formula $Q = 200\ M^{\frac{5}{6}}$, where

$Q$ = discharge in cubic feet per second, and

$M$ = area in square miles.

Burkli-Ziegler Formula $Q = R\ c\ \sqrt[4]{\frac{S}{A}}$, where

$R$ = average rate of rainfall during heaviest fall in cubic feet per second per acre,

$c$ = constant,

$S$ = general fall of area per 1000,

$A$ = area in acres.

Rational Formula $Q = A\ I\ R$, where

$A$ = area,

$I$ = imperviousness of ground,

$R$ = rate of rain fall.

Cooley's Formula $Q = 180\ M^{\frac{2}{3}}$, where

$M$ = drainage area in square miles.

As an aid to the use of these formulas for calculation of run-off, the run-off of several drainage districts in the Mississippi River valley has been computed, and compared with the actual capacity of the ditches or streams, which capacity is computed by the commonly accepted Chezy and Kutter formula.

## 2. Computation of Run-Off and Comparison with Actual Discharge.

### A. Lower Salt Fork Drainage District.

This district is located in the East Central part of Illinois. The land is flat, slightly rolling, and is practically all under cultivation. The Salt Fork empties into the Vermillion River near Danville. The general shape of the district is like that of a fan. The area of the district is 250 square miles.

Fanning $Q = 200 \times (250)^{\frac{5}{6}}$

$= 19800$ cubic feet per second.

Cooley $Q = 180 \times (250)^{\frac{2}{3}}$

$= 7860$ cubic feet per second.

Burkli-Ziegler $q = 1 \times 1.0 \sqrt[4]{\frac{10}{160000}}$ $R = 1$, $C = 1.0$, $S = 10$, $A = 160000$

$= 0.089$ cubic feet per second per acre,

$Q = 0.089 \times 160000$

$= 14100$ cubic feet per second.

Rational $Q = \frac{160000 \times 43560 \times 0.1 \times 0.5}{12 \times 60 \times 60}$ $R = 0.5$, $I = 0.1$

$= 8075$ cubic feet per second.

The ditch has a 70 foot bottom with 1 to 1 side slopes and a depth of 10 feet. By Kutter's formula, using the values

$s = 0.0002$, $n = 0.025$, and $r = 8.55$,

$v = 3.4$ feet per second,

$Q = A \; v$

$= 2905$ cubic feet per second, (actual capacity).

The actual capacity of the ditch is to the runn-off as computed by Fanning's formula, 14%; by Burkli-Ziegler's 20.6%; by Rational, 36.0%; and by Cooley's, 37.0%.

B. North Fork Special Drainage District.

This district is located in the Southern part of Illinois in Hamilton and Saline Counties. The land is flat and was swampy until drained. Run-off computations were made for three separate drainage areas all long and oval in shape, the areas being: 1, 50 square miles; 2, 70 square miles; and 3, 260 square miles. In the following work, let Q = run-off as computed by Fanning's formula, and $Q^1$ = run-off as taken care of by ditch according to Chezy and Kutter's formula.

Area 1.

$$Q = 50^{\frac{5}{6}} \times 200$$

$$= 5160 \text{ cubic feet per second.}$$

The ditch has a 16 foot base, $r = 5.0$, $s = .0007$, $A = 192$, $n = 0.025$, $c = 78.7$, $v = 4.65$.

$$Q^1 = 4.65 \times 192$$

$$= 894 \text{ cubic feet per second.}$$

$Q^1 = 17.3\%$ of Q.

Area 2.

$$Q = 70^{\frac{5}{6}} \times 200$$

$$= 6860 \text{ cubic feet per second.}$$

The ditch has $A = 224$ square feet, $r = 5.3$, $s = 0.0005$, $n = 0.025$, $c = 79.1$, $v = 4.07$.

$$Q^1 = 4.07 \times 224$$

$$= 913 \text{ cubic feet per second.}$$

$$= 13.3\% \text{ of Q.}$$

Area 3.

$$Q = 260^{\frac{5}{6}} \times 200$$

$$= 20300$$

The ditch has a 70 foot base, A = 624 square feet, r = 6.8, s = 0.0003, c = 83.0, and v = 3.75.

$$Q^1 = 3.75 \times 624$$

= 2340 cubic feet per second.

= 11.5% of Q.

## C. P. C. Knight's Examples.

P. C. Knight says: " I have found that the run-off of a valley having an area of 324 square miles (about 40 miles long and 8 miles wide) was carried without overflow by a channel having a bottom slope of 2.28 feet per mile and a cross-section of 650 square feet." In this case $Q = 324^{\frac{5}{6}} \times 200$

= 24570 cubic feet per second.

Assume this ditch to have a 75 foot base and 1 to 1 side slopes. Then A = 650 square feet, r = 6.4, s = 0.00043, c = 81.8, v = 4.31.

And $Q^1 = 4.31 \times 650$

= 2800 cubic feet per second.

= 11.4% of Q.

Knight also says: "In another case it took a channel having an area of cross-section of 94 square feet with a fall of 1.1 feet per mile, to carry the run-off of a valley of 8 square miles (approximately 4 miles long and 2 miles wide), the perimeter of which had an elevation of 20 feet above the bottom land." Assuming ditch to be 4 feet deep, A = 94 square feet, r = 3.0, s = 0.0002, c = 118.3, and v = 2.93.

$Q^1 = 2.93 \times 94$

$= 275$ cubic feet per second.

$Q = 200 \times 8^{\frac{5}{6}}$

$= 1130$ cubic feet per second.

$Q^1 = 24.4\%$ of Q.

D. Pekin and Lamarsh Drainage District.

This district is located on the Illinois River about ten miles below Peoria. It is protected from overflow by levees and the hill water is diverted, so is not taken into consideration. The shape of the district is almost square and the area is 4.14 square miles.

$Q = 200 \times 4.14^{\frac{5}{6}}$

$= 653$ cubic feet per second.

The main ditch has a bottom width of 8 feet, side slopes of 1 to 1, a maximum depth of 6 feet (assumed), and a slope of 1 in 5000. Then $r = 3.36$, $s = 0.0002$, $c = 82.0$, $v = 2.12$ feet per second.

$Q^1 = 2.12 \times 84$

$= 178$ cubic feet per second.

$= 27.3\%$ of Q.

E. Red River Valley in North Dakota.

The Red River valley is very flat and in most parts has a slope of only 1 to 3 feet per mile. From data given out in the United States Department of Agriculture, Bulletin 189, it is possible to compare results and design with other districts.

1. Forest River in the Salt Lake District.

Forest River drains an area of 644 square miles. It has side slopes of 1 to 1, bottom width of 35 feet, grade of 8 feet

per mile, and a depth of 9 feet. Therefore, r = 6.56, c = 81.5, s = 0.0015, and v = 8.13 feet per second.

$Q^1$ = 8.13 X 396

= 3210 cubic feet per second.

Q = 200 X $644^{\frac{5}{6}}$

= 43700 cubic feet per second.

$Q^1$ = 7.4% of Q.

2. Willow Coulee District.

The Willow Coulee drains an area of 167 square miles. It has side slopes of 1 to 1, a bottom width of 24 feet, grade of 1.5 feet per mile, and a depth of 9 feet. Therefore, r = 6.02, c = 81.3, s = 0.00035, v = 3.72 feet per second.

$Q^1$ = 8.72 X 297

= 1110 cubic feet per second.

Q = 200 X $167^{\frac{5}{6}}$

= 14240 cubic feet per second.

$Q^1$ = 7.8% of Q.

3. Drayton District. (Main A)

Main A drains 34 square miles, has side slopes of 1 to 1, grade of 1 foot in 1 mile, a bottom width of 8 feet, and it is 8 feet deep. Therefore, r = 4.18, c = 76.3, s = 0.00019, and v = 2.15 feet per second.

$Q^1$ = 2.15 X 128

= 275 cubic feet per second.

Q = 200 X $34^{\frac{5}{6}}$

= 3830 cubic feet per second.

$Q^1$ = 7.2% of Q.

F. Cook Ditch in Kankakee River Valley.

From data in United States Department of Agriculture, Circular 80, the area drained is 14,140 acres, the bottom width is 8 feet, 1 to 1 side slopes, and a grade of 3.2 feet per mile. Assuming depth of 8 feet, r = 4.18, c = 75.9, s = 0.0006, and v = 3.82 feet per second.

$Q^1$ = 489 cubic feet per second,

$Q = 200 \times 22.1^{\frac{5}{6}}$

= 2635 cubic feet per second.

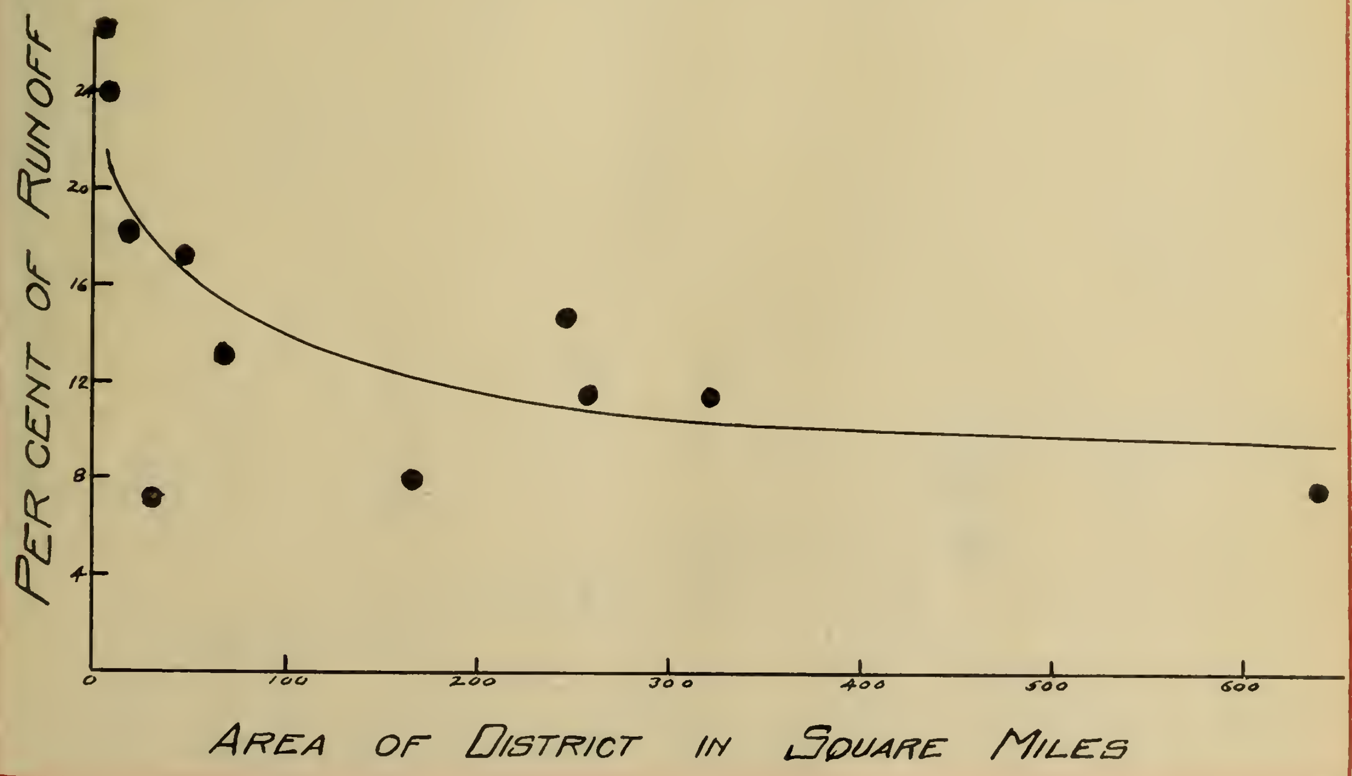
$Q^1$ = 18.5% of Q.

In order to compare the results obtained above, they are put in tabular form below.

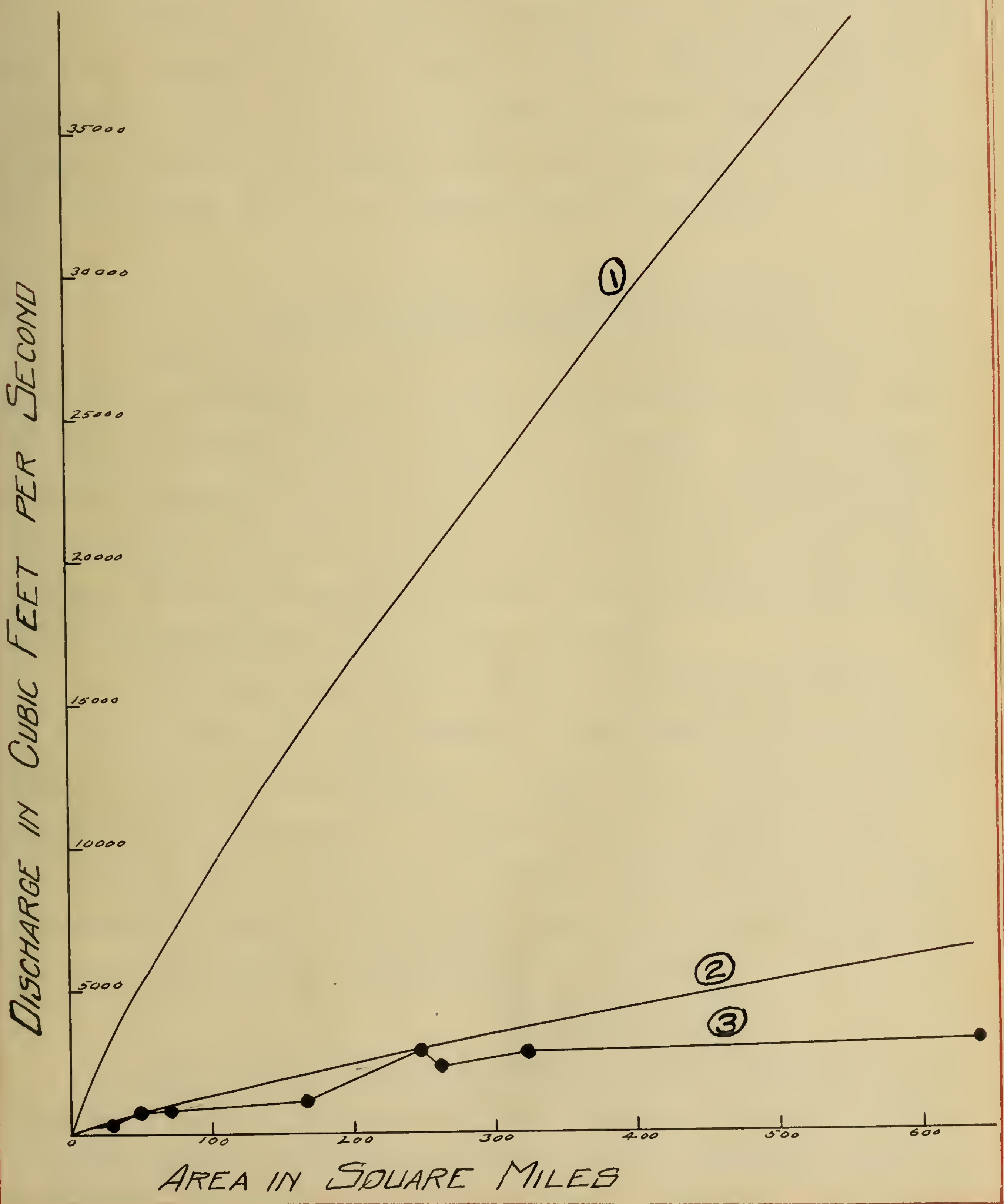
| Drainage District as classified above. | Area in Square Miles. | Q as defined above. | Q expressed in inches per day. | Q' as defined above. | Q' expressed in inches per day. | Ratio of Q' to Q expressed as per-cent. |
|---|---|---|---|---|---|---|
| E 1 | 644 | 43700 | 2.54 | 3210 | 0.185 | 7.4 |
| C 1 | 324 | 24570 | 2.84 | 2800 | 0.321 | 11.4 |
| B 3 | 260 | 20300 | 2.93 | 2340 | 0.334 | 11.5 |
| A 1 | 250 | 19800 | 2.97 | 2905 | 0.431 | 14.7 |
| E 2 | 167 | 14240 | 3.20 | 1110 | 0.247 | 7.9 |
| B 2 | 70 | 6860 | 3.67 | 913 | 0.484 | 13.3 |
| B 1 | 50 | 5160 | 3.87 | 894 | 0.664 | 17.3 |
| E 3 | 34 | 3840 | 4.22 | 275 | 0.305 | 7.2 |
| F 1 | 22.1 | 2635 | 4.47 | 489 | 0.821 | 18.5 |
| C 2 | 8 | 1130 | 5.29 | 275 | 1.276 | 24.4 |
| D 1 | 4.1 | 653 | 5.91 | 178 | 1.601 | 27.3 |

From United States Department of Argiculture Bulletin 230, the formula used in designing ditches in the St. Francis' Valley Drainage project in Northeastern Arkansas, was $C = \frac{24}{M} + 6$, where C = run-off in cubic feet per second per square mile. This gives results smaller than the ones obtained above, and indicates that the above results are probably in error on the side of safety. These above results seem to show that the run-off, as computed, by Fanning's formula, is too large. It is noted with satisfaction that the relation of $Q^1$ to Q seems to vary with the size of the area. The districts included in the above are all examples of good practice. They are doing the work satisfactorily, and although in widely different parts of the country, they seem to indicate that the conditions for maximum rain fall and run-off, are about the same over the entire Mississippi Valley.

The graph below shows the relation of the size of the district to the per cent of computed run-off as compared to Fanning's formula run-off, which has been used in the above examples.



The graph below shows the relation between the area and the discharge; first, by Fanning's formula, second, 15% of Fanning's formula, and third, by the capacity of the ditches.



A formula of the type of Fanning's or Cooley's is undoubtedly the most reliable. While much depends upon the soil and the general slope of the ground, it is known that the condition of land which is in need of drainage is in many cases similar. For large districts this is especially true. The Burkli-Ziegler formula and the Rational formula permit a greater degree of refinement and they are sometimes used for small areas. However, for large areas there is too great a chance for error.

3. Run-Off as Affecting Design. (A, Ditches) .

The function of the ditches in a drainage district is to remove all surplus water, so as to avoid injury to crops. It is seen, therefore, that the run-off as discussed above bears a direct relation to the capacity of the ditches. The discussion above concerns maximum floods and run-off, so if the ditches are designed for these maximum conditions, they will easily take care of the under-drainage in an ordinary rain fall

It is advised that the ditches be designed to dispose of a run-off which is a per centage of that computed by Fanning's formula or some similar formula. A graph such as shown above is a great aid in doing this. At the same time it should be kept in mind that such a graph is for average conditions and that the topography and shape of the area as well as the annual rain fall in that section of the country should be considered.

B, Pumping Plant.

The pumping plant in a drainage district should in general be designed to dispose of the run-off as expressed above plus the seepage. If this is not done, we are making our whole

interior of ditches inefficient. As has been said, there is a seepage to be pumped out. In some pumping districts seepage is very small in amount, while in others it amounts to considerable. It should be provided for in any case. Many engineers make a practice of adding a constant amount, or, of multiplying the run-off as computed, by a constant greater than 1. This is good practice and makes a convenient way of handling the proposition.

Where there is under-drainage, say 3 feet, there is considerable amount of storage reservo r. It is proper to utilize this storage space and the capacity of the pumping plant may be modified to some extent. The ditches can be used as reservo rs, and in case they occupy considerable area, quite a saving in pumping machinery can be made.

To illustrate the way in which run-off affects the design of ditches and pumping plant, let us take the example of the Coal Creek Drainage and Levee District in Schuyler County, Illinois. This district has an area of about 11 square miles and the tile drains are laid at an average depth of 3 feet 6 inches, over the entire district. From graph we get Q = 317 cubic feet per second. The ditch should be designed to carry this amount at station 0. The pumping plant data was taken as follows, assuming 1 inch rain fall per 24 hours.

| | |
|---|---|
| 43560 | = feet in 1 acre |
| 7000 | = acres |
| 12 /304,920,000 | = square feet in district |
| 25,410,000 | = cubic feet per 24 hours |
| 7.5 | |
| 4/190,575,000 | = gallons per 24 hours |
| 47,643,750 | = amount absorbed, evaporated, etc. 1/4". |
| 2/142,931,250 | = gallons remaining to be pumped |
| 24/71,465,625 | = gallons that will rea h pump in 24 hours |
| 2,977,776 | = gallons per hour. |

| | | |
|---|---|---|
| 30,000 | = | pump capacity in gallons per minute |
| 60 | | |
| 1,800,000 | = | " " " " " hour |
| 24 | | |
| 43,200,000 | = | " " " " " day |
| 21,416,200 | = | actual storage capacity of ditches in gallons |
| (64,616,200 | = | total capacity of district to receive and take off water. |
| (71,465,625 | = | gallons that will reach pump in 24 hours. |

In this actual case it is seen that the pumping plant has a capacity of 66-2/3 cubic feet per second, or 21% of run-off as estimated conservatively. This district has proved to be efficient, that is, the pumping plant does its duty efficiently, so this may be said to be good practice in designing pumping plants. The figures given above are those used in the design for that district.

# Chapter III.

## Ditches.

### 1. Formulas for Discharge of Ditches.

The common formulas for discharge of ditches, take the general form $v = c\, r^x\, s^z$, where c is a constant, "r" the ratio of the area of cross-section to the wetted perimeter or hydraulic radius, and s the slope or gradient of the ditch.

The most common formula for velocity of flow in ditches was first proposed by Chezy. It is $v = c\sqrt{r\, s}$, where c is a coefficient, r is the hydraulic radius, s is the slope, and v is velocity in feet per second. Kutter's formula for the determination of c in the Chezy formula is in almost universal use. The value of c is expressed in terms of the hydraulic radius r, the slope s, and the degree of roughness of the surface n. It is

$$c = \frac{\frac{1.811}{n} + 41.65 + \frac{0.00281}{s}}{1 + \frac{n}{\sqrt{r}}\left(41.65 + \frac{0.00281}{s}\right)}$$

, in which n is an abstract number which depends only upon the roughness of the surface. The values of n assigned by Kutter were: n = 0.020 for canals in very firm gravel, n = 0.025 for canals free from stones and weeds, n = 0.030 for canals and rivers with some stones and weeds, and n = 0.035 for canals and rivers in bad order.

As the result of measurement of discharge made under his direction, Professor A. N. Talbot gives, n = 0.025 for larger ditches, and n = 0.030 for smaller ditches.

C. W. Brown, C. E., uses n = 0.030 for interior ditches, and n = 0.020 for open ditches. These results have been checked several times and found to give values nearly correct.

Elliott has proposed a formula which has been used to a large extent for areas of one thousand acres or more. It is of the same general form as Chezy's, being:

$Q = a\sqrt{\frac{a}{p} \times 1\frac{1}{2} f}$ in which Q = discharge in cubic feet per second, a = cross-section of water way, p = wetted perimeter, and f = fall in feet per mile.

In the engineering record of November 4, 1911, Professor C. T. Johnston, and R. D. Goodrich write as follows: "On account of the unwieldy form of Kutter's formula, exponential equations of the form $v = c\ r^p\ s^q$ have been suggested by several engineers to give velocities in canals and pipes, the values of the exponents p and q being determined so as to make c practically a constant for all values of r and s and dependent only upon the character of the channel. As a result of a series of investigations and measurements of velocity in a considerable number of existing ditches and canals, which varied widely as to materials, grade, condition of repair, etc., the following table is given: q = 0.5.

| | c | p |
|---|---|---|
| Clean straight ditch in earth or firm gravel free from vegetable growth, and made smooth by use | 59.8 | 0.76 |
| New ditch in earth with straight channel | 45.5 | 0.80 |
| Old ditch in earth with straight channel and vegetable growth | 42.8 | 0.83 |
| Old ditch in earth with crooked channel and some vegetable growth | 35.0 | 0.85" |

This formula is very new and nothing is known of its application, except what was gathered from the article.

## 2. Cross-section of Ditches.

The problem of designing a cross-section of a ditch is

one involving many factors. The solution is by the cut and try method. The proper cross-section in any case is one which will give the required capacity, with a given slope, and which will also give a desired velocity. The problem of determining the slope and velocity will be taken up later on and, for the time being, the cross-section will be considered alone.

In the formula $v = c\ r^x\ s^p$ the cross-section affects the factors c and r. The most economical form of cross-section from a stand point of loss of head due to friction is the semi-circle. The form adopted in practice is trapezoidal. The slopes of the sides of the ditch are to be determined by considering the character of the soil. The crumbling nature of the exposed soil, occasioned by the action of the frost, requires the side slope to be not steeper than 1 to 1, if reasonable permanence is to be expected. A 1 to 1 slope will be flat enough for all ordinary soils, however for sandy or loose materials a 1-1/2 to 1 slope will be safer. This 1-1/2 to 1 slope will stand up fairly well if the soil is at all firm. The method of construction should be considered in determining the side slopes. If a dipper dredge is to be used, it will build a ditch with steep slopes, and the bottom should be made wider to allow for the caving which is sure to follow.

The width of berm is a matter which should receive careful attention. The idea in having a berm on a ditch is to make it impossible for the removed earth to slide back into the ditch. A width sufficient to prevent this should be specified, but the width of berm should not be made unnecessarily large. E. E. Watts in the Engineering News of February 13, 1902, says: "A practice which has been favorably received is to design the width of berm

equal to 1/2 the base, where cuttings do not exceed 6 feet. For heavier cuttings this ratio should be increased 1/2".

The relation of depth of ditch to width is an indeterminate problem. It should be borne in mind that a cross-section approaching as nearly to the semi-circle as possible will reduce the loss of head due to friction to a minimum. C. E. Grunsky in the Engineering News of September 10, 1908, says as follows: "As a general safe guide, but remembering that a wide range of departure from standard is allowable, the following relation of depth to width on the water surface may be accepted: Calling water surface S and depth d,

| | | | |
|---|---|---|---|
| For | Ws = 6 | make d = | 1.5 |
| | = 10 | = | 1.6 |
| | = 20 | = | 1.5 |
| | = 40 | = | 3.3 |
| | =100 | = | 6.7 |
| | =200 | = | 9.0 |
| | =300 | = | 11.0" |

It is a wise precaution to insist that ditches be cut a little below grade as drainage ditches have a marked tendency to fill up immediately after construction.

### 3. Slope of Ditches.

In the formula for velocity for ditches $v = c\ r^p\ s^q$, the slope or gradient affects the factors $s^q$ and c. It is obvious that the velocity increases as the slope increases. The determination of the slope of ditches should be given careful thought. The slope is determined in most cases by the topography, as it is found to be best economy to make the slope of ditches as near to parallel to the general slope of ground as possible. In this way

the limiting slope is determined. It should be remembered that the ditches are to be an outlet for the tile drains, and due allowance must be made for this in studying the grade for the ditches. The slope and cross-section determination go together, of course, and the main requirement is that they shall have required capacity without giving too great a velocity.

As a matter of interest in this connection an article by E. E. Watts, in the Engineering News of February 13, 1902, is quoted as follows: "The writer has constructed ditches seven to ten miles in length, through loam and sand, and on a subsoil of fine gravel with a slope of 1 in 4220, and work has been in successful operation for a number of years. The allowable minimum in this character of soil is 1 in 6000. Through black loam in a clay subsoil with a like cross-section, and similar length, another work was constructed with a slope of 1 in 1780. The allowable minimum in this kind of soil is probably 1 in 7500. In black muck and quick sand the writer has information of work having been attempted with a slope of 1 in 8000 which proved a failure. In this character of soil the minimun should be 1 in 3000. (The ratios are based on a cross-section area of 225 square feet)".

## 4. Velocity in Ditches.

The question of the proper velocity of flow in drainage ditches is a much mooted question. Engineers have written many papers from time to time, some advocating the use of high velocities so as to keep the ditch clean, and as many others have written favoring low velocities. There are practically no drainage ditches

which have not filled up or been eroded to a certain extent. The ideal velocity is one that will keep the ditch clean by its scouring action, but which does not have enough scour to wash the banks out. This condition is practically impossible to realize, for the reason that conditions of flow at different times of the year are totally different, and also it is a difficult matter to determine what velocity is desired for any certain soil. C. E. Grunsky in the Engineering News of September 10, 1908, says: "The velocity at which water will be allowed to flow without eroding the bottom of earth channels is usually from 2 to 3 feet per second, and for ordinary conditions can be taken at 2.5 per second." High velocity when admissible is always desirable because then there will be no silting up of the channel. The proper velocity is largely dependent upon the character of the soil, and must be determined for every district separately.

J. W. Dappert in the 1906 Report of the Illinois Society gives the following table showing scouring velocities:

| Character of soil. | Velocity in feet per second. |
|---|---|
| For fine clay | 0.40 |
| For very fine sand | 0.70 |
| For light vegetable soils | 0.83 |
| For sand as course as flax seed | 0.91 |
| For gravel size of pea | 1.40 |
| For small pebbles | 2.52 |
| For pebbles 1 inch in diameter | 3.00 |

This table may be of value, but nothing could be learned of its derivation or its application. The more conservative practice has been to use rather low velocities in the design of ditches.

It has been found that a well designed ditch will not fill up with silt rapidly, and the ditch designed for low velocities, say 1.5 to 2 feet per second, will be most economical in the long run.

## 5. Location and Construction of Ditches.

As in the case of slope of ditches, the alignment of ditches is dependent upon the topography of the drainage district. The cut will be less and the construction cheaper if they are made to follow the natural depressions and streams, if any. The curves should have a large radius and should be made similar to the natural curves in streams in the near vicinity.

The use of dredge boats in the construction of drainage ditches has become quite general of late years. The popularity of the dipper dredge is deserved because it is, in many cases, the cheapest form of construction. Where care is taken a good clean looking ditch can be built with a dipper dredge. The engineer should look to it, to see that a machine with a long enough boom is used. In many cases this is not done, the result being that the berm is not made large enough and much of the material slips back into the ditch. If it does not slip back, it causes pressure on the bank and causes the bank to slide.

It is some times wise to use teams and scrapers for small ditch work. When a ditch is built in this way, the banks are likely to be too flat. This form construction is used very little now, excepting where it is impossible to use a machine of some sort. Ditching machines of the Austin type build an excellent ditch and altho rather expensive, their use may be economical in the long run.

This type of machine can build and has built the neatest and best ditches in operation today. Drag-line dredges are coming into more general use, because of the clean ditch which it is possible to construct with them. The banks of the ditch made with this machine are rather steep, but a sufficient berm can be left, and generally the ditches constructed with the drag-line dredge are very sightly.

## Chapter IV.

## Tile Drains.

### 1. Duty of Tile Drains Determined by Run-Off..

The problem of the design and location of tile drains is one which, in many respects,is similar to the problem of designing ditches. The same consideration determine the amount of water which is to be removed in a day. These are briefly: Character of soil, topography, shape of district, climate, distribution of rain fall, and ability of various crops to withstand injury from excessive soil moisture. The run-off to be taken care of our drainage systems was discussed fully in Chapter II, and the duty of the tile drains is a little more than their proportional part of the run-off if anything. It is obvious that the size of the drainage unit makes no difference at all in determining the duty of the tile drains, and if the run-off as discussed in Chapter II is used as a basis of determining their duty, the run-off, should be computed for a very small acreage. It has been very general practice in designing tile drains to assign a certain amount to be

removed in 24 hours, but the selection of this amount arbitrarily is to be discouraged. Rules given by different writers are given below, but they are necessarily very crude. Quantity of water to be removed by tile drainage:

| Authority. | Remarks. | Inches on surface per day. |
|---|---|---|
| Debauve | | 0.262 |
| Herve-Magnon | | 0.258 |
| Friedrich | | 0.105 |
| Wage and Mollendorf | Compact soil | 0.160 |
| Wage and Mollendorf | Open soil | 0.228 |
| Schweder | Clay | 0.238 |
| Schweder | Light soil | 0.196 |
| Le Clerc | | 0.22 to 0.27 |
| Elliott | Good drainage | 0.50 |
| Elliott | Ordinary drainage | 0.25 to 0.33 |

## 2. Spacing and Depth of Tile Drains.

Having chosen the depth of water which we will drain per day, our problem is to make a design and location of tile drains, such as will give us effective drainage, as economically as possible. As has been stated before, the object of drainage is to keep the ground water plane below the depth to which the roots of plants will penetrate for moisture, oxygen, and food. Oxygen is essential to plant life, and it is obvious that the roots can obtain no oxygen below the ground water plane. This depth for the successful cultivation of most agricultural products is about 36 inches; a greater depth will be more effective, while fairly good results are sometimes obtained with a depth of 24 inches or less below the surface of the ground. It has been found impossible, even with the aid of elaborate mathematical analysis, to determine the precise expression for the form of ground water between two lines of drains. Mr. Robert Horton in the Michigan Engineer for 1906 has given the problem of proper depth and spacing

of tile drains a very elaborate treatment. He assumes it to be sufficiently accurate to treat the curve between two drains as a parabola. Then letting it be required to lower the ground water table to a depth (h) feet below the ground in N days, he derives the following expressions:

$$W = H\ I \ /\ D$$
$$D = H\ I \ /\ W$$
$$W = \frac{N\ H\ I}{4\{P - Z\}(H - 2h)}$$
$$D = \frac{4(P - Z)}{N} \times (H - 2h)$$

where P = percentage of voids in soil, D = average depth of water in inches on surface to be removed per day, H = depth of drains below surface in feet, h = depth to which it is required that the ground water table mid-way between drains shall be lowered in M days after saturation, Z = average per centage of moisture remaining in soil above ground water table expressed decimally, and I = transmission constant, or depth of free downward free filtration in the soil in inches on the surface per day. To obtain the value of I he has plotted a diagram from Slichter's formula giving I in terms of effective diameter of the soil particles, also a set of temperature corrections. He says briefly; "I wish to emphasize the fact that the character of the soil, rather than the size of the drain controls the rate and efficiency of the drainage. The first step in designing a drain system is to determine the water yielding capacity of the soil. This being given the proper depth and spacing of the drains can be determined therefrom, and from considerations of quantity of water to be removed per day, etc., after which the necessary size of drains can be determined by the usual methods. Strangely enough, none of the

authors of books on land drainage seem to take into proper consideration the character of the soil as affecting the efficiency of drainage. ********** The relation of these physical laws to the problem is quite as delicate as the laws govening the design of a dam or a dynamo, and no one without a knowledge of underlying principles is competent to design one or the other, be he farmer, county surveyor, or civil engineer".

S. M. Woodward in United States Agricultural Department Bulletin No. 243, says: "Parallel lines of five or six inch tiles are laid at such distances apart as the elevation, slope, and nature of soil require. Where lines are long, larger sizes are found necessary in the lower portions. The distances between the parallel lines found necessary by trial are usually between 10 and 20 rods, and their minimum depth below the surface should not be less than 30 inches".

The writer believes that the nature and character of the soil should be taken into consideration in determining the amount of water to be drained in 24 hours. The imperviousness of the soil and the amount of capillary water retained by the soil influence greatly the effectiveness of the drainage. These same factors should be considered in determining the depth and width of the drain. It is obvious that the character of the soil mainly determines the effective slope of the ground waters toward the drains. This effective slope can be determined for a certain soil by experiment, and the problem of the proper depth and width apart of the drains is a simple matter. For instance let us assume the effective slope in some particular instance to be 1 in 50.

Let the width between drains be 400 feet, then to give effective drainage to a depth of 3 feet in the center, the drains must be at a depth of 7 feet. The need of test to determine this effective slope is great, and until these are made, it is necessary to make experiments in each case to make a proper determination of the distance apart and depth of the sublaterals. This is a very unsatisfactory way to leave this subject, but in our present state of ignorance concerning the effective slopes of ground water in different soils, there are not enough real facts known to give us a more definite basis to work on.

## 3. Formula for Discharge of Drain.

Having determined the spacing and depth of the tile drains and their duty, it is next necessary to consider the size of tile to be used. The formula for velocity of flow in tile or pipe lines, which is in most general usage is the Chezy and Kutter formula. It is $v = c\sqrt{r\ s}$, and

$$c = \frac{\frac{1.811}{n} + 41.65 + \frac{0.00281}{s}}{1 + \frac{n}{\sqrt{r}}\left(41.65 + \frac{0.00281}{s}\right)}$$

in which s = slope, r = hydraulic radius, and n = coefficient of roughness.

J. F. Rightmore and M. Chappel made a set of experiments at Iowa State College in 1910 to determine the value of n in Kutter's formula. The obtained the following results:

| Size | Material | Character of Gradient | Depth of Flow | n |
|---|---|---|---|---|
| 2'X 32" | Cement | 0.01 to 0.05 | 1/2" to 3/4" | .01504 |
| 2' 34" | " | Regular | 7-3/4" to 9-1/2" | .01638 |
| 2' 20" | " | " 0.3 to 0.6 | 4" to 7-1/2" | .01146 |
| 2' 18" | Vit.Clay | Ir-" 1.0 to 2.0 | 1-1/2" to 3" | .01172 |
| 2' 14" | " " | Ir-" 0.15 to 1.1 | 1-1/4" to 2-1/2" | .01525 |
| 1' 14" | " " | 1.2 to 1.7 | 1/2" to 1" | .01683 |
| 1' 10" | Clay | Irregular | 1/2" to 1" | .01640 |

The 20 inch and 18 inch tiles were best laid and that seems to affect the value of n more than any other factor does.

In Engineering Contracting of October 25, 1911, E. J. Parker says: "Several formulas have been proposed for flow in tile, the one most used probably being the one known as Poncelot's formula. it is $v = 48\sqrt{\frac{d\ f}{l - 54\ d}}$ , where d = diameter of tile in feet, f = total fall of line in feet, l = total length of line in feet, and v = velocity in feet per second. Recent experiments have shown that this is inaccurate, and that Kutter's formula using n = 0.014 gives results more nearly correct for drains of good construction."

The slope being determined, the proper size of tile to carry a given run-off can be determined by means of these formulas. Of the formulas, probably either one is sufficiently accurate for ordinary purposes, but Kutter's formula seems to be preferred by a majority of engineers.

## 4. Slope of Tile Drains.

As in the case of ditches, the slope of tile drains is limited or affected to a degree by topographical considerations. In many cases we have no choice in the matter of slope whatever, the elevation of the source and outlet being fixed. Where possible deep cuts should be avoided, for after a certain limiting depth is reached, the expense of making the cut increases in much

greater proportion than the depth of cut. Tables are often published giving capacities, minimum grades, and lengths for given slopes and sizes of tile. If these tables are used, great care should be taken. In general it is much preferable to make computations for the individual cases. Many engineers heartly condemn the use of tables at all, as their use leads to mistakes so easily.

## 5. Location and Construction of Tile Drains.

As a matter of economy of construction, tile lines should be run in the natural depressions of the ground. The problem of the arrangement of tile drains is a special one and cannot be taken up here. In general it may be said that the best system will be that one which has the least overlapping of different drains. The question of allowing for loss of head at intersections and for loss of head due to curvature is one which has been neglected in the past, and B. J. Parker in Engineering Contracting of October 25, 1911, has a very able paper on those subjects. He says: "The following table is to be used as a guide in proportioning drop at the inter section. It is the practice of the writer to join the inverts and make up the drop in the first ten or twenty-five feet of the entering lateral, the longer distance being used for the larger sizes."

| Diameter in inches. | Drop in feet. | Diameter in inches. | Drop. |
|---|---|---|---|
| 5 | 0.2 | 18 | 0.8 |
| 6 | 0.3 | 20 | 0.9 |
| 8 | 0.4 | 24 | 1.0 |
| 10 | 0.45 | 30 | 1.2 |
| 12 | 0.5 | | |
| 14 | 0.6 | | |
| 16 | 0.7 | | |

## Conclusion.

It is known that there are many other problems coming up in the design of large draiange districts, which cannot be handled in a thesis of this length. The mechanical engineering problem concerning the pumping plants in pumping districts would be a good topic for discussion. The question of drainage assessment is one about which there is considerable agitation,and unfair assessments have been one great drawback to reclamation work over the entire country. The legal problems, the writing of specifications, and many other things are topics upon which there is need of intelligent discussion.With the state and national drainage conventions,which have been started in late years, bringing these questions to the attention of capable men, it is believed that a better day for reclamation by drainage is near at hand.